Seismic Performance of Gravity Quay Walls in Unimproved and in Improved Soils

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ABSTRACT

Recently strong earthquakes around the world caused serious disasters for port structures. It shows the shortcoming of current design and construction procedures and the needs to develop the performance based guidelines for seismic design. The performance-based seismic design method has taken in the seismic design standards of buildings and bridges. However, port structures still use working stress design method. There are lots of improvements needed to be done to upgrade the current design codes. The objective of this research is directed towards numerically investigating the seismic behaviour of gravity quay walls/soil system. For this purpose, 2D nonlinear dynamic analyses of soil–structure system are carried out with the aid of finite difference software, FLAC. The investigation has been also examined the effectiveness of ground improvement techniques in controlling permanent seismically-induced deformations to provide guidelines for remediation optimization. Based on the results, design considerations have been provided.

1. INTRODUCTION

Experience at ports has demonstrated that waterfront retaining structures are highly susceptible to earthquake-induced damage. A significant percentage of the damage is due to the liquefaction of adjacent soils (PIANC, 2001; Iai, 2005).

This research focused on numerical analyses. The authors have utilized a numerical model, validated with well-documented case histories, for parametric studies of caisson performance in unimproved and in improved soils. The results of this study have been synthesized into practice oriented design charts for estimating the lateral deformations of gravity quay walls.

2. NUMERICAL MODEL

The computer program FLAC (Itasca, 2005) was used in this study. The numerical modeling has been analyzed under plane strain and fluid–mechanical interaction conditions. The soil modeling was performed using an effective stress based Mohr-Coulomb constitutive model. For the quay wall, the concrete caisson has been modeled by linear elastic elements, with soil–structure interfaces modeled by joint interfaces.

3. SEISMIC PERFORMANCE EVALUATION CHARTS

Parametric Study Using a Structurally Simplified Model

The factors governing the seismic performance of a caisson-type quay wall include wall dimensions, the thickness of the soil deposit below the wall and liquefaction resistances of subsoil below and behind wall, as well as the levels of seismic shaking at the base layer. In this study, the soil deposit below the wall was represented by a sand backfill used to replace the original soft clay deposit in order to attain the required bearing capacity.

The standard cross section of the caisson used for the parametric study is shown in Figure 1. Major cross-sectional dimensions were specified by the width (W) and the height (H) of the caisson wall and by the thickness of subsoil (D1) and thickness of the backfill (D2).

In this study, the peak acceleration of the input seismic excitation assigned at the base layer is the north-south direction of Kobe vertical seismic array. This motion is scaled to six different acceleration values ranging from 0.1g to 0.6g. The thickness of the liquefied soil deposit below the wall (D1) was specified by a ratio with respect to the...
wall height (H), ranging from D1/H = 0.0 (i.e. a rigid base layer located immediately below the wall) to D1/H = 1.0 (i.e. thick soil deposit below the wall).

![Fig. 1: Typical Cross-section of Gravity Quay Wall for Parametric Study](image)

The thickness of the liquefied soil deposit behind the wall (D2) was specified by a ratio with respect to the wall height (H), ranging from D2/H = 0.0 (i.e. a non liquefied soil layer) to D2/H = 1.0 (i.e. a liquefied soil layer with depth equal to the height of the wall).

Other geometrical conditions assumed for the effective stress analyses include: wall height H = 13m, water level = 2 m lower than the top of the wall, thickness of the rubble mound = 4m.

The geotechnical conditions of the soil deposits below and behind the wall were assumed to be the same, represented by the equivalent SPT-N value (the corrected SPT-N value for the effective vertical stress of 65 kPa of an equivalent relative density. The SPT-N value used in this study were SPT-N = 5, 10, 15, 20, 25 in unimproved soil.

**Parameter Sensitivity and Charts**

The results of the parametric study were summarized in terms of the residual horizontal displacement (d) at the top of the wall. The residual horizontal displacement was normalized with respect to the wall height (H). The effects of the major parameters on the normalized residual horizontal displacement (d/H) are discussed as follows.

**Width-To-Height Ratio (W/H)**

As seen from Figure 2 at SPT N = 5 increasing the width to height ratio (W/H) leads to significant decrease in the normalized lateral displacement (d/H), especially for W/H = 1.05 and 1.02 and at higher input excitation level 0.4g, 0.5g and 0.6g. Similar trends are shown in Figure 3 and Figure 4 for SPT N = 10 and 15 respectively. However, in Figure 5 at SPT N =20 and Figure 6 at SPT N= 25 increasing the width to height ratio (W/H) leads to negligible effect on the normalized lateral displacement (d/H) at any excitation level for different soil conditions (i.e W/H = 1.05 is considered as the optimum).

**Input Excitation Level (g)**

Increasing the input excitation level lead to significant increase of the lateral normalized displacements (d/H) and normalized vertical displacements especially for the SPT N values of 5 and 10.

**SPT-N Value**

The effects of the equivalent SPT N value are shown in Figures 2-6 for width-to-height ratio (W/H) 0.50, 0.65, 0.90, 1.05 and 1.20. For W/H=0.50 and W/H=0.65 increasing the SPT N value leads to decrease the normalized lateral displacement and it can be shown clearly for higher input excitation level 0.4g , 0.5g and 0.6g. This trend is the same for W/H = 1.05 and 1.20.
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Fig. 5: Effects of the Width-to-Height Ratio W/H on Normalized Lateral Displacement (d/H) (for SPT N value of 20) at D1/H=1.0

Fig. 6: Effects of the Width-to-Height Ratio W/H on Normalized Lateral Displacement (d/H) (for SPT N value of 25) at D1/H=1.0

Fig. 7: Effects of Thickness of Soil Deposit below the Wall on Normalized Lateral Displacement (d/H) (for SPT N value of 5) at W/H=0.90

Fig. 8: Effects of Thickness of Backfill on Normalized Lateral Displacement (d/H) (for SPT N Value of 5) at W/H=0.90 and D1/H=1.0

**Thickness of Soil Deposit below Wall (D1/H)**

The effects of the thickness of the soil deposit below the wall is shown in Figure 7 for W/H=0.90. There is a linear relation between D1/H and d/H as D1/H reaches 0.5 the rate of change greatly increases particularly for g above 0.4. This comment is the same for different N values up to 25 which clarify that the thickness of a soil deposit below the wall and its SPT N values are two important factors that affect the displacement.

The thickness of the soil deposit below the wall significantly affects the displacement. For a wall built on a thick soil deposit (D1/H> 0.50), the effects of the equivalent SPT N values of the soil below and behind the wall are significant.

**Thickness of Backfill Soil (D2/H)**

The effects of the thickness of backfill soil behind the wall is shown in Figure 8 for W/H=0.90 and at D1/H=1.0. When the level of excitation is high, significant increase in the displacement is recognized for D2/H> 0.50 and for smaller SPT N values, this clarify that the existence of a soil deposit behind the wall and its SPT N values are two important factors that affect the displacement.

**Parametric Studies of Improved Soil**

The next phase of this research focused on an extensive parametric study using the structurally simplified model for gravity quay walls as shown in Figure 1. Caisson height of 13 m was used with width-to-height (W/H) ratios of 0.65 and 0.90 respectively. The foundation and backfill soils are cohesionless materials, each layer modeled with uniform density. The range of densities used can be correlated with stress corrected standard penetration resistances SPT-N of (a) 25 in the foundation soil, (b) 5, 10 and 15 in unimproved backfill, and (c) 30 in improved backfill. The soil improvement was modeled as providing a uniform increase in soil density throughout the zone of treatment. In this study, the peak acceleration of the input seismic excitation assigned at the base layer is the north-south direction of Kobe vertical seismic array at depth -32m. This motion is scaled to 3 different acceleration values 0.2 g, 0.4 g, 0.6 g.

**Analysis Results**

The results of the parametric study demonstrate the influence of ground motion characteristics, geotechnical
parameters, and caisson geometry on the deformations of the retaining walls. These results have been synthesized into normalized parameters, where possible, to incorporate the key variables into straightforward design parameters. For example, the wall geometry has been expressed by W/H ratios as previously mentioned, the width of the zone of soil improvement is given as a function of the height of the wall (L/H).

The results of the parametric study are shown from Figure 9 and Figure 10. The normalized lateral deformations at the top of the wall, d/H, are plotted versus the normalized width of the improved soil, L/H, and as a functions of backfill density and the W/H ratios of the caissons. In these figures, the rubble fill adjacent to the caissons has been treated as non-liquefiable soil, thereby contributing to the “effect” width of the improved (i.e., non-liquefiable) soil. The relationships provided in Figure 9 and Figure 10 clearly demonstrate the benefit of ground treatment on the seismic performance of the caissons. It was also evident that the incremental benefit of a wider zone of ground treatment begins to decline once the soil improvement extends more than about 2.0 to 3.0 times the total height of the wall. At this point the cost of additional soil improvement may outweigh the benefits. It is interesting to note that the soil improvement guidelines prepared by the PHRI (1997) correspond to a normalized width of soil improvement of roughly 1.3 to 1.6, as supported by the work of Iai (2005). Also Dickenson and Yang (1998) demonstrated that the incremental benefit of a wider zone of ground treatment begins to decline once the soil improvement extends more than about 2.0 to 3.5 times the total height of the wall.

4. CONCLUSIONS

This investigation had highlighted the following pertinent aspects of the seismic design and performance of caisson type quay walls.

1. The proposed nonlinear, effective stress analysis has been proved that it is a more appropriate method to simulate the seismic performance of the caisson as compared to conventional design approaches. It is more efficient especially when excess pore pressure generation and liquefaction have occurred in the backfill.

2. Among the parameters considered in this study, the most sensitive parameter affecting the quay wall displacement under a prescribed level of shaking was the range of SPT-N values of subsoil below and behind the wall. Second was the thickness of backfill soil (D2/H). Thirdly was the thickness of the soil deposit below the wall (D1/H). Fourthly was the width to height ratio (W/H).

3. Improving the engineering properties of the backfill reduces the wall deformation. The recommended treated zone range from L/H= 2.0 to 3.0 for caisson of W/H range from 0.65 to 0.90, respectively.

REFERENCES


